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DESIGN OF GEOSYNTHETICALLY REINFORCED EMBANKMENTS USING DECOMPOSED GRANITE AS BACKFILL MATERIAL

by

Jorge G. Zornberg, Nicholas Sitar and James K. Mitchell

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ABSTRACT

This report presents a summary overview of the results of a study of the feasibility of using geosynthetic reinforcements in construction of embankments with decomposed granite backfill. Results from triaxial tests on specimens from the Shasta Bally Batholith show that the peak friction angle of compacted decomposed granite is adequate and it is a function of the confining stress, decreasing from about 46° at 50 kPa to about 38° at 1300 kPa for material compacted to 90% relative density. The results of a centrifuge study of the performance of geosynthetically reinforced embankments at failure validate current design practices. However, refinements to current procedures are suggested where appropriate, based on the findings of the centrifuge study. Cost evaluation for the construction of geosynthetically reinforced embankments shows that they compare favorably with other types of slope construction. A case history of a geosynthetically reinforced embankment built using decomposed granite shows that the construction can be rapid and can be readily achieved with regular field equipment. Thus, overall, there seem to be no technical impediments to the use of decomposed granite materials in the construction of geosynthetically reinforced embankments

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1 Introduction

The purpose of this report is to provide a summary overview of the results of a study of the feasibility of using geosynthetic reinforcements in construction of embankments with decomposed granite backfill. Design guidelines for reinforced soil structures using decomposed granite as backfill material are presented. The design of geosynthetically reinforced soil embankments is based on conventional limit equilibrium analyses, adapted to take into account the stabilizing effect of the forces generated in the reinforcements. Although based on its grain size distribution decomposed granite may satisfy current backfill requirements for geosynthetically reinforced embankments, some considerations should be made to accommodate the special character of this material (Yapa et al., 1993).

Centrifuge testing has provided much needed evidence that limit equilibrium methodologies adequately predict the performance of geosynthetically reinforced soil structures at failure (Zornberg et al., 1995). Although these results validate current design practices in the United States (e.g., Christopher et al., 1989), refinements to current procedures are suggested when appropriate based on the findings of the centrifuge study. This report addresses slopes on firm foundations, and assumes that the slopes are permanent structures.

2 Use of reinforcements in earthwork construction

Design and construction of stable slopes and retaining structures within limited right-of-way are aspects of major economical significance in geotechnical engineering projects. When geometry requirements dictate changes of elevation in highway projects, the engineer faces a variety of distinct alternatives for the design of the required earth structures. Traditional solutions have been either the design of concrete retaining walls (Figure 1a) or of conventional embankment slopes (Figure 1d). Concrete retaining walls (either gravity or cantilever) have been the conventional choice for many projects involving construction under the constraints of limited access. Although simple to design, standard wall alternatives have generally led to elevated construction and material costs, that often constitute a significant fraction of total project bids. The traditional alternative to concrete retaining walls has been the use of unreinforced slopes. However, the construction of conventional embankments, often with flat slope angles dictated by conventional stability analyses, is precluded on projects in which design is controlled by space constraints.

Soil reinforcement, which involves the use of inclusions in a soil mass to improve its mechanical properties, has become a widely used earthwork construction method that provides technically attractive and cost-effective grade separations at the ground surface. Reinforced soil walls (Figure 1b) generally provide vertical grade separations at a lower cost than do traditional cast-in-place concrete construction. Ribbed steel strips, steel bar mats, geogrids, and geotextile sheets, are examples of typical reinforcement elements. Reinforced wall systems additionally involve the use of shotcrete facing protection or of facing elements such as precast or cast-in-place concrete panels. Alternatively, steepened reinforced slopes (Figure 1c) may eliminate the use of facing elements, thus saving material costs and construction time in relation to the vertical reinforced wall. The use of reinforced slopes often constitutes the most cost-effective solution in highway projects involving the addition of traffic lanes within the right-of-way of existing embankments.

The decision-making process for selecting an earth structure involves a trade off between the imposed space constraints and the construction costs of the retaining structure. The optimum design alternative is to be defined by project-specific conditions, however, the general trends are as shown in Figure 1. Depending on the available right-of-way, the figure illustrates that the optimum alternatives for projects involving grade separations are reinforced soil walls and reinforced soil slopes. As indicated by dashed lines in the trends suggested in the figure, both conventional and reinforced retaining walls require equivalent right-of-way and that both conventional and reinforced slopes often result in equivalent construction costs.

The use of inclusions to improve the mechanical properties of soils dates to ancient times. However, it is only within the last quarter of century or so (Vidal, 1969) that analytical and experimental studies have led to the contemporary soil reinforcement techniques, used for a wide range of earthwork construction. Soil reinforcement is now a highly attractive alternative for embankment and retaining wall projects because of the economic benefits it offers in relation to conventional retaining structures. Moreover, its acceptance has also been triggered by a number of technical factors, that include aesthetics, reliability, simple construction techniques, good seismic performance, and the ability to tolerate large deformations without structural distress.

The performance of geotextile-reinforced soil structures involves many complex soil-structure interactions which defy simple characterization. Current knowledge of most aspects of reinforced soil behavior stem from a combination of testing and modeling that support current design procedures (Jewell, 1993). Testing of the reinforcements, of the backfill soil, and of the interactions between them provides the parameters needed for design. However, it is through numerical modeling, physical modeling, and the instrumentation of field structures, that we are coming to understand the principles of soil reinforcement and the mechanisms that characterize the behavior of reinforced soils structures.

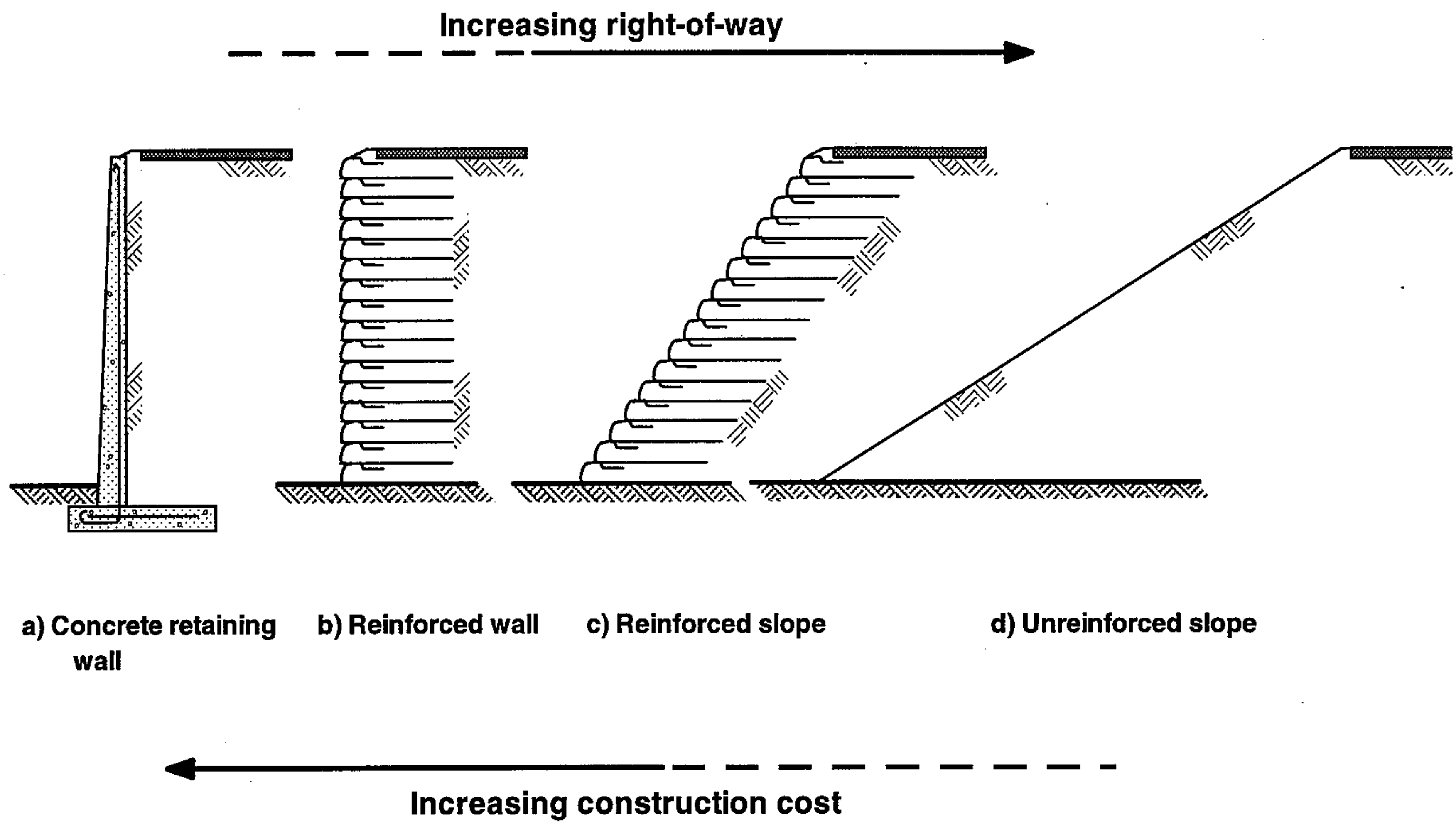


Figure 1 - Alternative designs for earth retaining structures

3 Decomposed granite as backfill material

The term "decomposed granite" is used to identify a broad range of materials, from slightly weathered but intact rocks and boulders, to sand, and even clay. The in situ fabric of decomposed granite typically contains coarse grained aggregate of minerals, predominantly quartz, weathered feldspar and mica, and a relatively small proportion of fines. Extensive evaluation of the mechanical properties of decomposed granite from the Shasta Bally batholith in northern California, was performed using oedometer, triaxial, direct shear, and pullout tests by Yapa et al. (1993). Figure 2 shows the grain size distribution for decomposed granite obtained from Shasta County. Decomposed granite particles undergo substantial breakage under relatively low loads. Experimental results show that breakage is primarily controlled by the applied strain level rather than the stress level. Shear-induced breakage in conventional triaxial tests is greater than in oedometer tests, probably because of the greater shear stress/strain component under triaxial conditions.

The friction angle of compacted decomposed granite decreases significantly with increasing stress level. In dense triaxial specimens, under confinements ranging from 100 to 1500 kPa, the reduction in ϕ_{peak} value was about 25 percent. The results of a series of triaxial ICD (Isotropically Consolidated Drained) tests on samples from the Shasta Bally batholith obtained by Yapa et al. (1993) are shown in Figures 3 (specimens at 90 percent relative compaction) and 4 (95 percent compaction). The top half of each figure shows the variation of σ_1/σ_3 with axial strain under confining pressures ranging from 100 to 1500 kPa. The bottom half of each figure shows the variation of the corresponding volumetric strain. Increased confining pressure clearly reduces the peak shear strength, suppresses shear-induced dilation of the compacted specimens, and increases the axial strain level at which the peak shear strength is attained. The peak angle of shear resistance (ϕ_p) of each of the specimens is plotted in Figure 5 against the corresponding confining pressure. The reduction in ϕ_p with confining pressure is substantial.

Even though an evaluation of the variation of strength properties with confining pressure is critical for the analysis of high embankments, generally the stability will be governed by the strength parameters of the backfill at reasonable low confining pressures if the embankments are on firm foundations. Consequently, additional triaxial ICD tests were performed as part of this study in order to define the strength parameters of decomposed granite at lower confining pressures than those investigated by Yapa et al. (1993). Moreover, the variability of the mechanical properties of decomposed granite from different batholiths was evaluated. A number of triaxial ICD tests were performed on samples of decomposed granite obtained from Idaho batholith and compacted at 90% RC. Figure 6 shows the variation of σ_1/σ_3 with axial strain and the corresponding variation in volumetric strain for the additional tests. The variation of the peak angle of shear resistance with the confining pressure is plotted in Figure 7 for the range of confining pressures varying from 50 to 400 kPa. For dense specimens of the Shasta Bally batholith ϕ_p decreased from 55° to 42° within this range of confining pressures. For loose Shasta Bally specimens, at 90% RC, ϕ_p was approximately 4° lower than for the dense specimens. In comparison, peak friction angles obtained from the Idaho specimens at 90% RC are in turn 4° lower than the values obtained from Shasta Bally specimens, also at 90% RC and equivalent confining pressures. Thus, these results show that a significant variability of strength results should be expected for different sources of decomposed granite, stressing the need for determining site-specific strength parameters.

The performance of decomposed granite in pullout tests and shear interface strength tests under high normal pressures was performed to evaluate the variability of geosynthetic-soil strength parameters under probable fill placement and environmental conditions (Yapa et al., 1993). The pullout coefficient of interaction of a geogrid embedded in dense decomposed granite decreased by more than 50 percent when normal pressure was increased from 70 to 700 kPa. In direct shear (decomposed granite-geogrid) interface strength tests, $\phi_{residual}$ values were nearly equal to those from direct shear tests of the soil alone.

Settlement and hydrocompression in oedometer specimens were not large under axial pressures as high as 1600 kPa, probably because breakage in these specimens was small. However, decreasing the compaction water content significantly increased the hydrocompression.

Based on these findings, it is recommended that compaction water content in a fill be maintained near the optimum. Compaction density should be decided upon after comparing the economics of alternative designs that consider steepness of the slope, utility of geosynthetics and acceptable settlement.

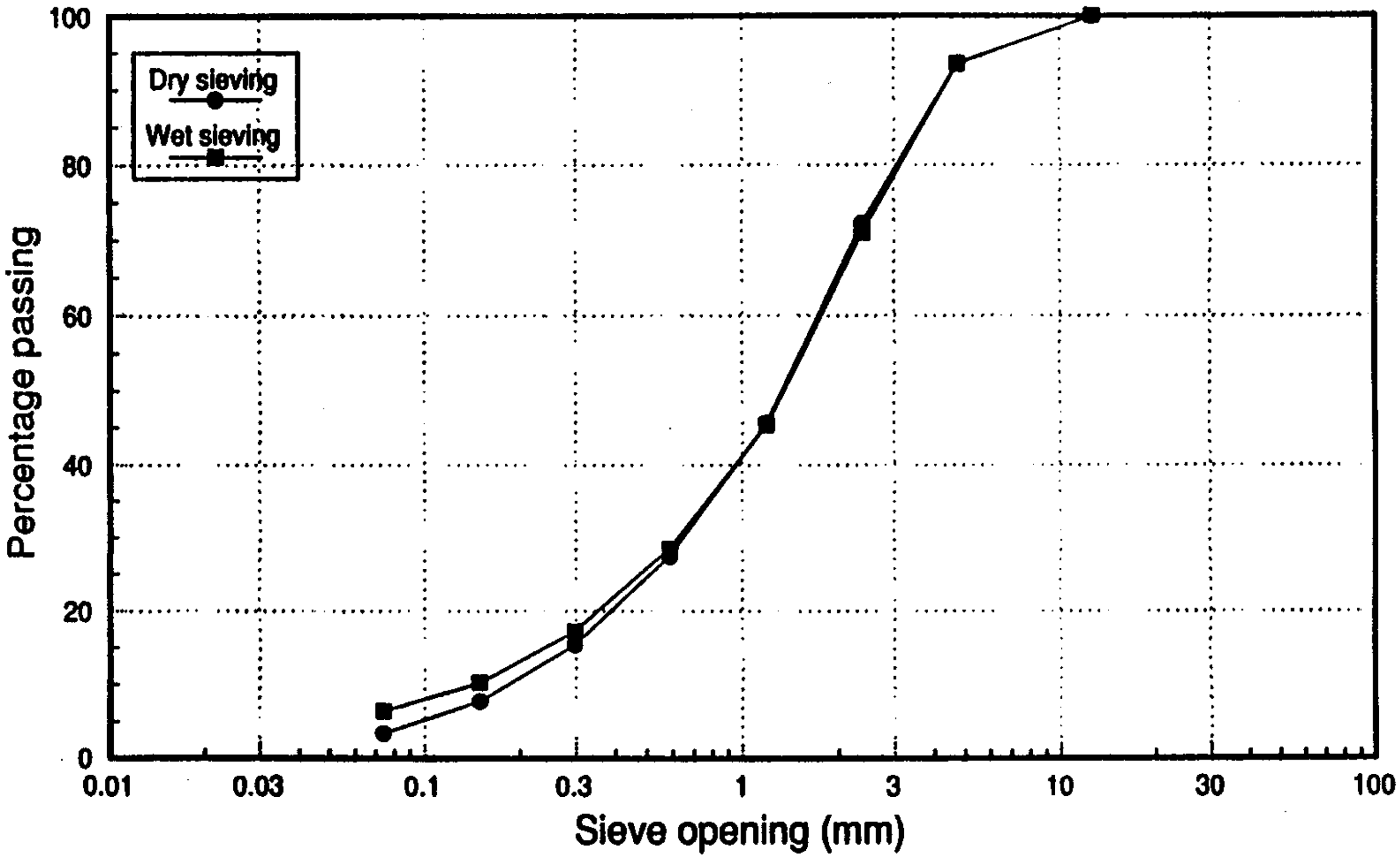


Figure 2 - Grain size distribution for decomposed granite from Shasta Bally Batholith (after Yapa et al., 1993)

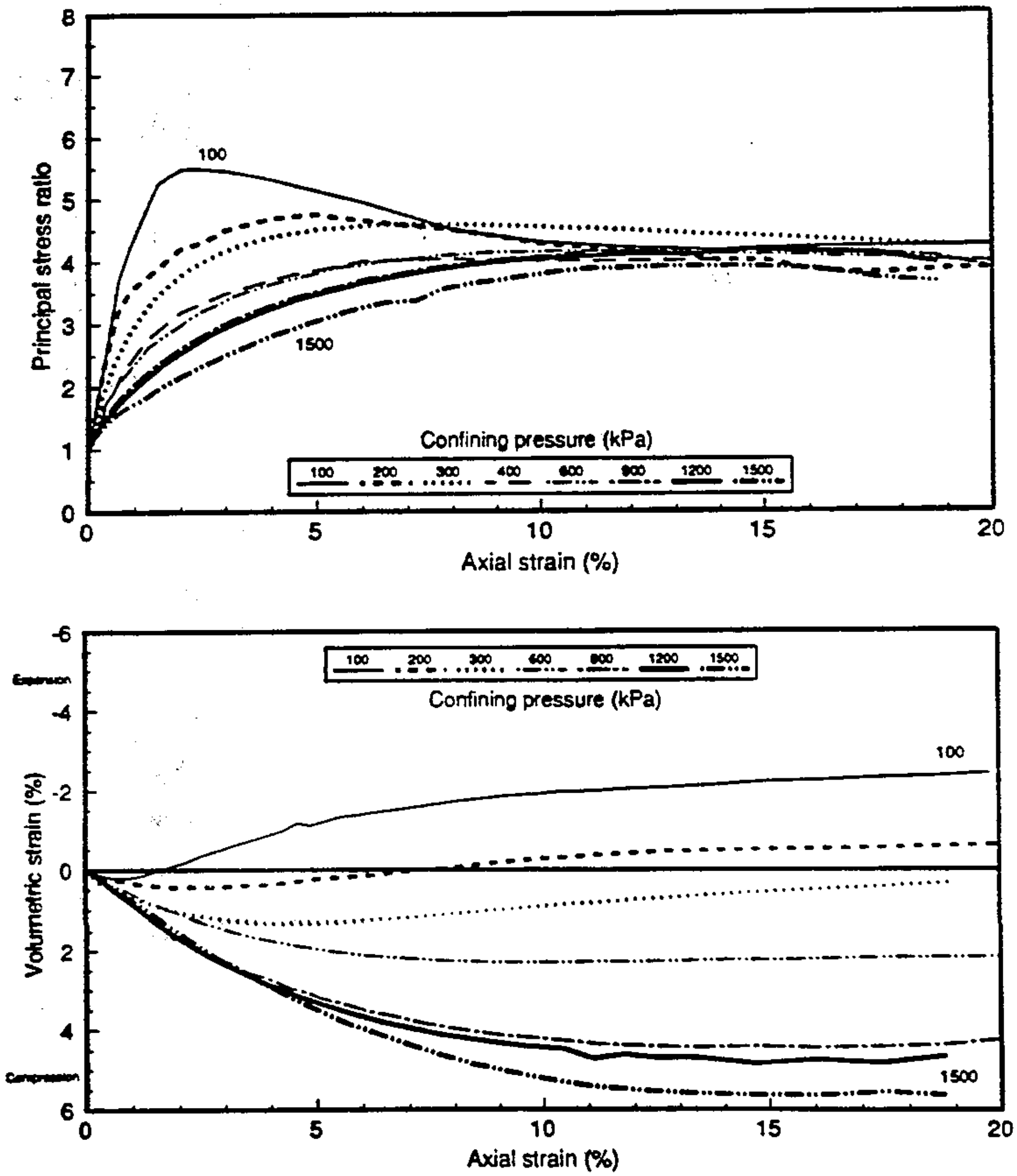


Figure 3 - Effect of confinement on σ_1/σ_3 and ε_v of triaxial specimens of decomposed granite from the Shasta Bally Batholith, RC = 90% (after Yapa et al., 1993)

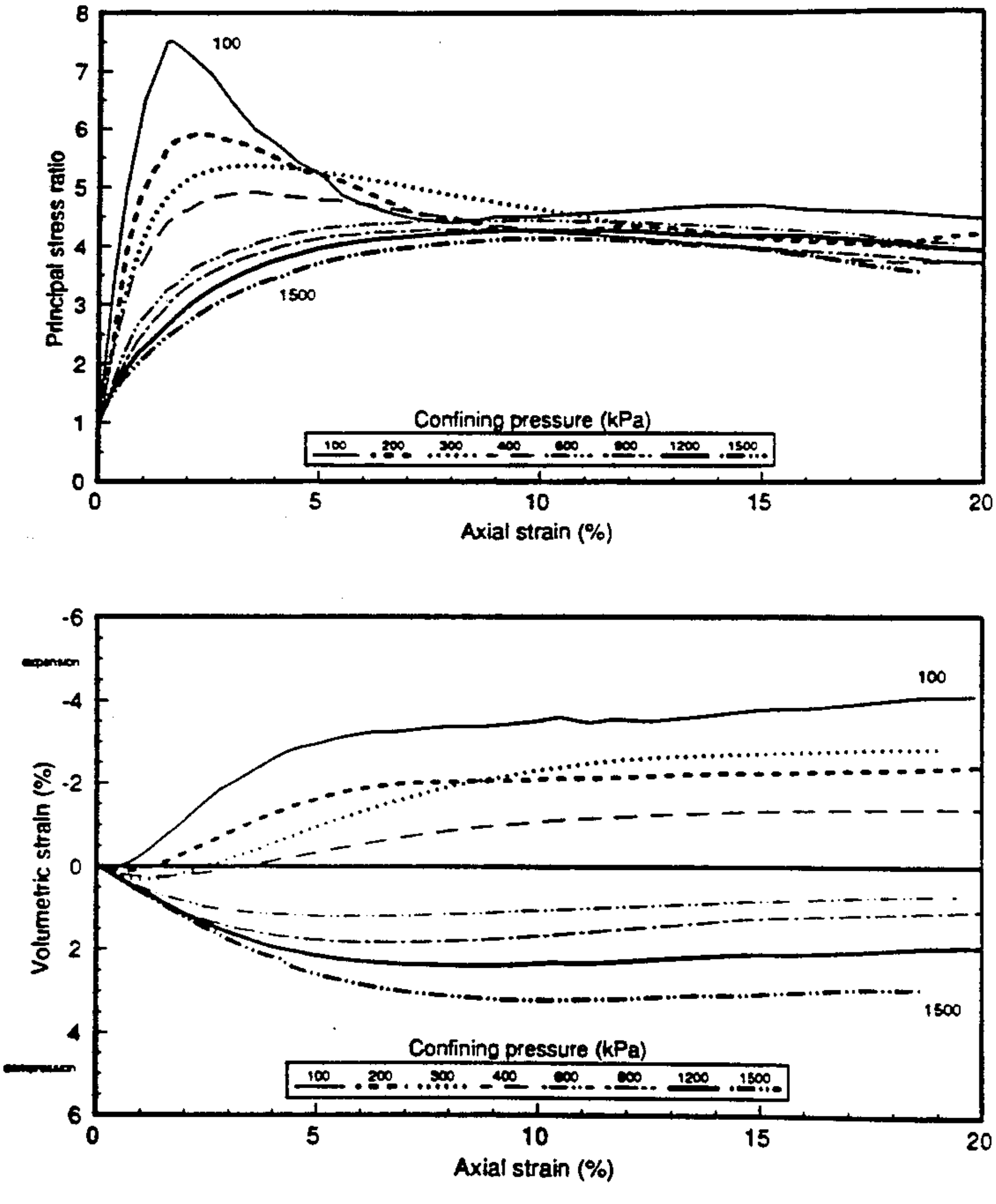


Figure 4 - Effect of confinement on σ_1/σ_3 and ε_v of triaxial specimens of decomposed granite from the Shasta Bally Batholith, RC = 95% (after Yapa et al., 1993)

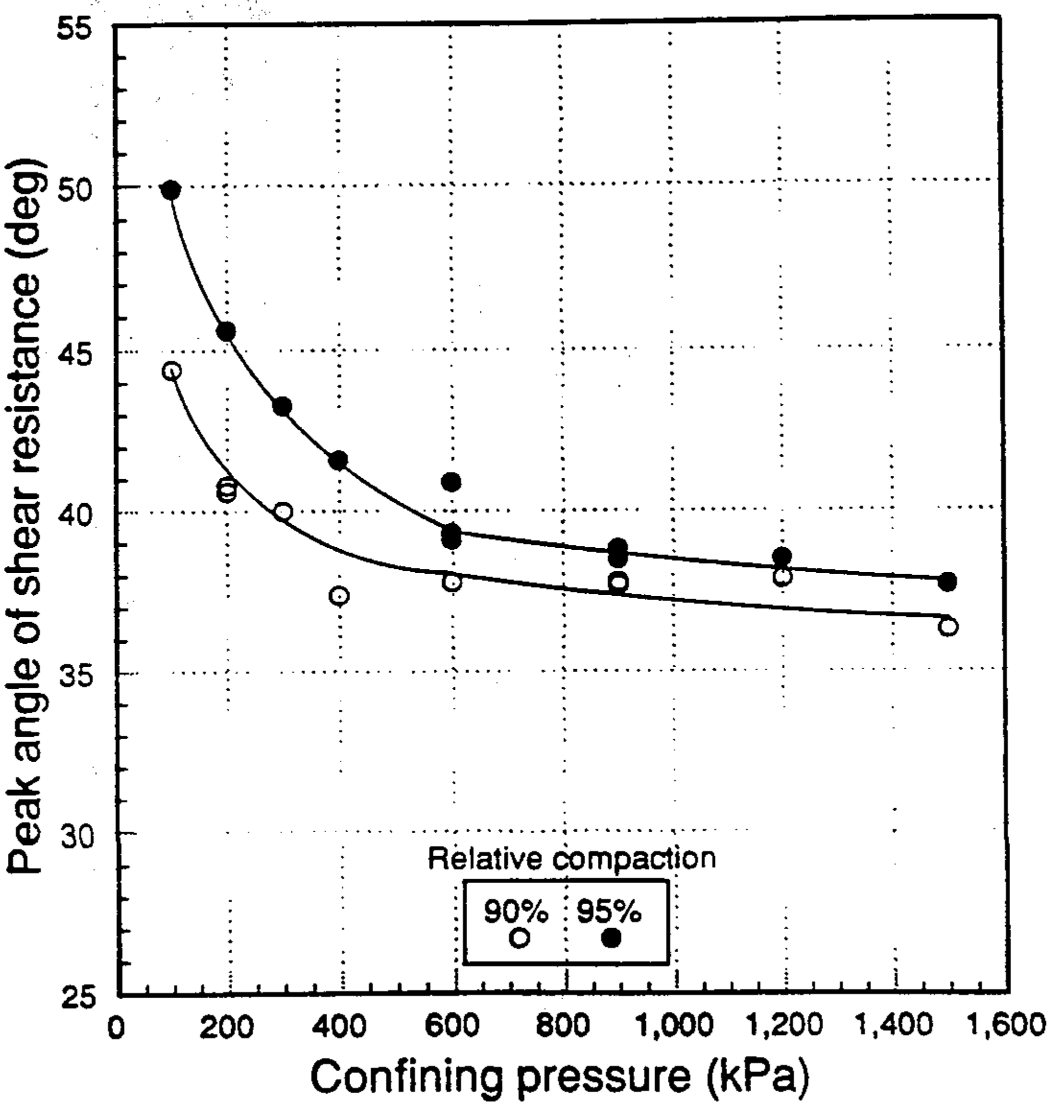
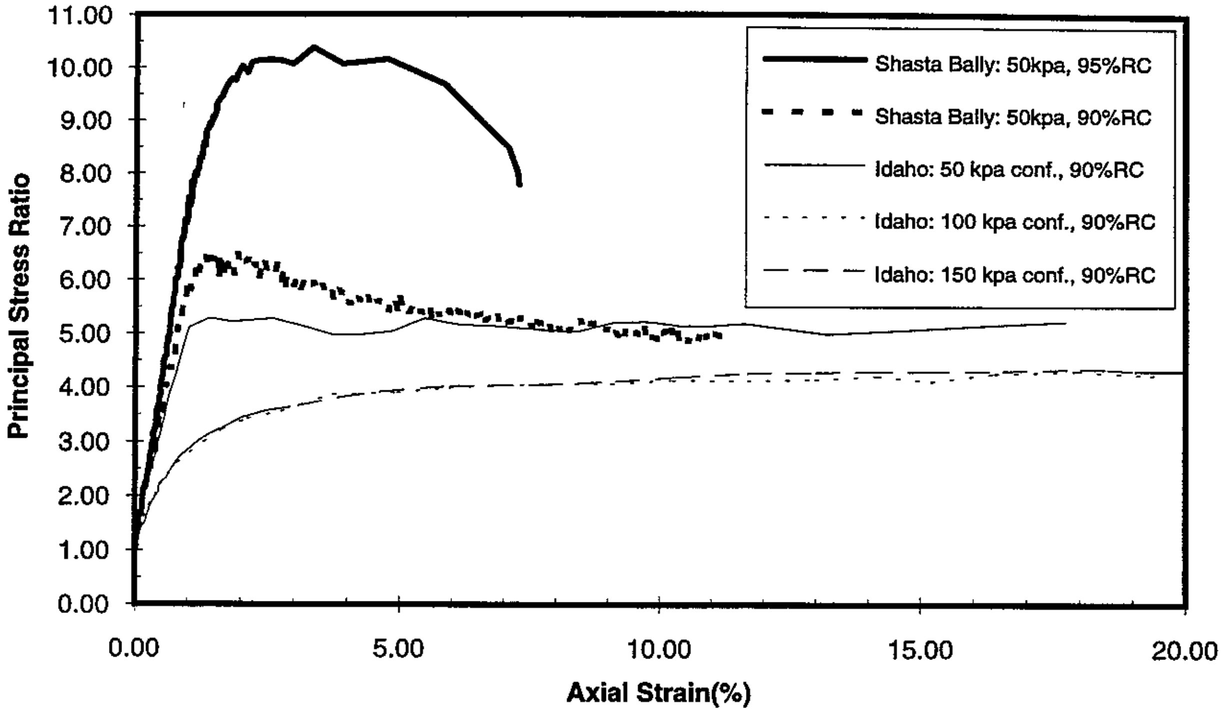


Figure 5 - Variation of ϕ_f with confining pressure in decomposed granite from the Shasta Bally Batholith (after Yapa et al., 1993)



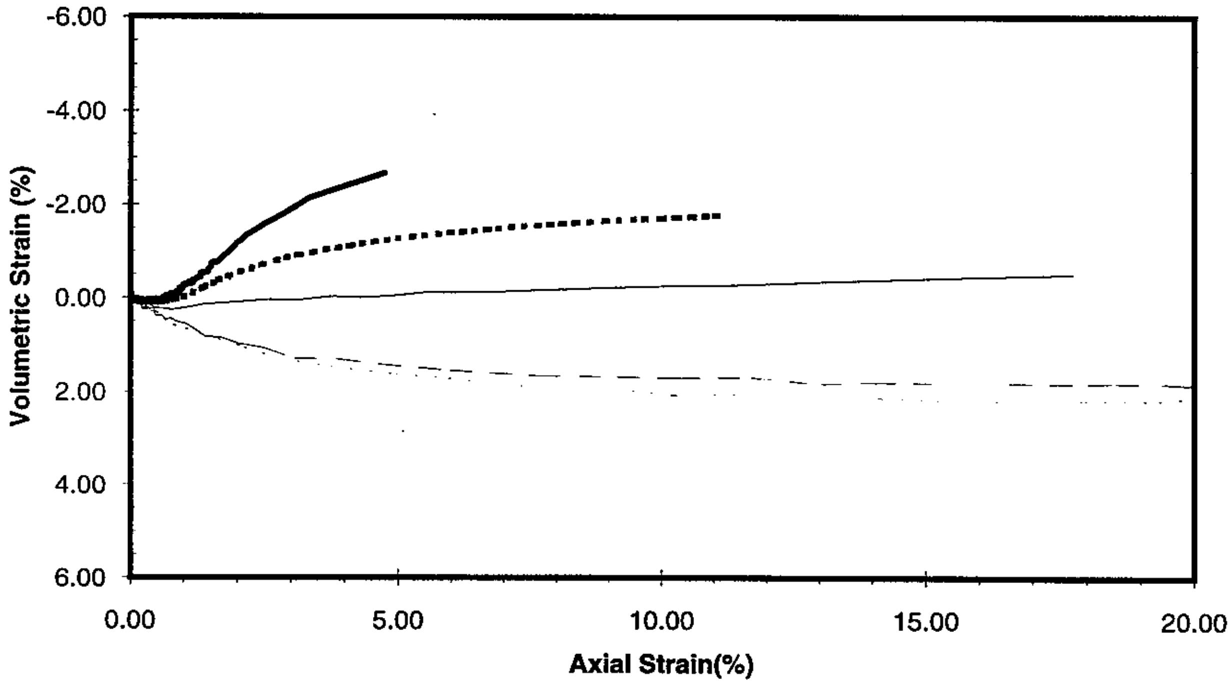


Figure 6. Effect of confinement on σ_1/σ_3 and ε_v of triaxial specimens of decomposed granite from the Shasta Bally and Idaho Batholiths

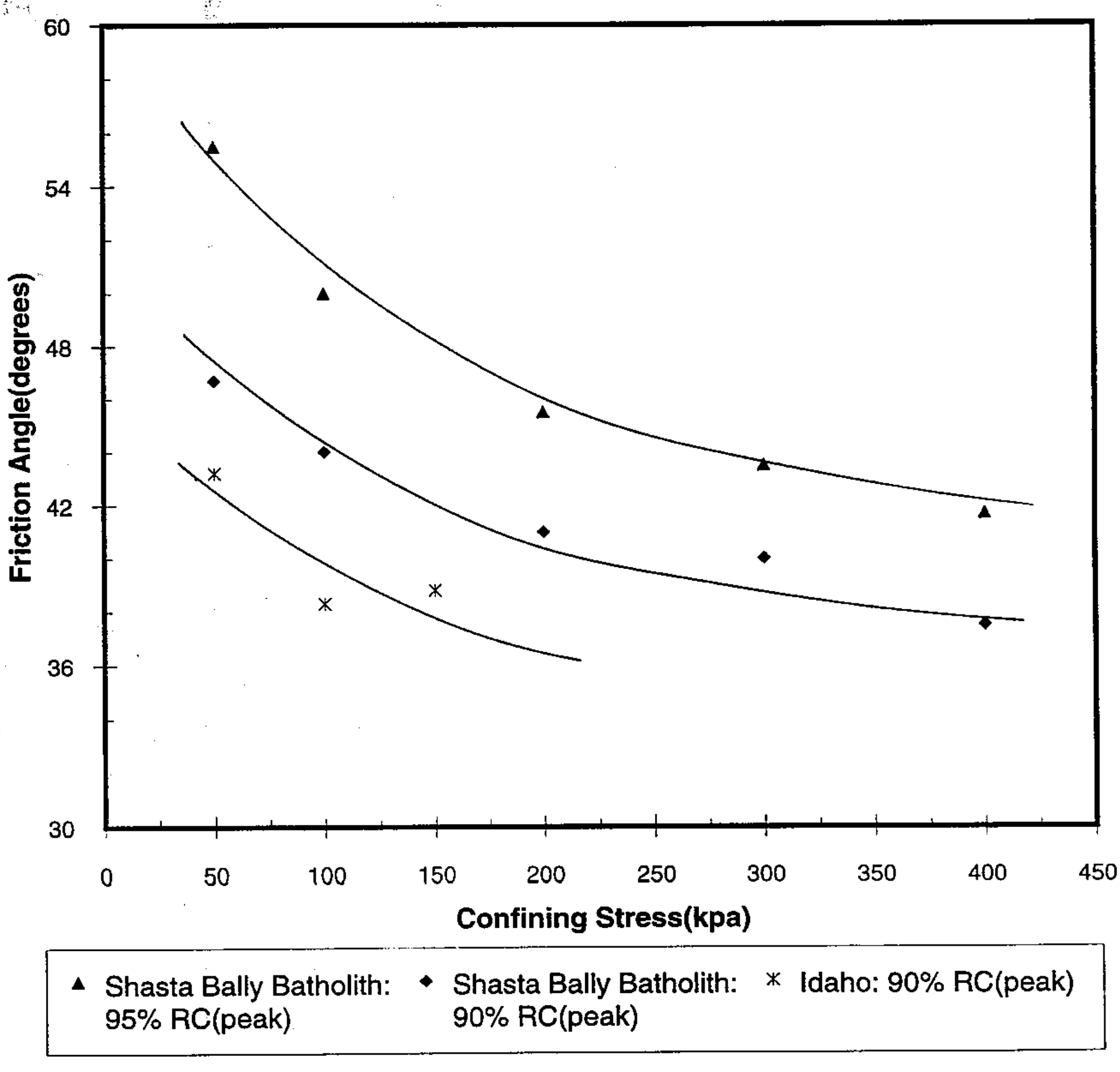


Figure 7 - Variation of ϕ_r with confining pressure in decomposed granite from the Shasta Bally and Idaho Batholiths

4 Use of geotextiles as reinforcements

As the availability of suitable construction sites decreases, there is an increasing need to utilize poor soils for foundation support and earthwork construction (Mitchell, 1981). Although the different soil reinforcement systems have greatly extended the use of soil as construction material, their use has often been limited by the availability of good-quality granular material which has generally been specified for the backfill. Even though there are several reasons for requiring good quality granular backfill, this specification has restricted the use of reinforced soil structures in cases where such material is not readily available. Undoubtedly, substantial cost savings and new soil reinforcement applications would result if indigenous fine grained soils as well as appropriately treated industrial and mine wastes could be used as backfill materials.

Steel has been the most widely used reinforcement material, however, the possibility of corrosion of these reinforcements is high, and has precluded their use in certain applications. With the introduction of polymer geotextiles and geogrids, non-corrosive reinforcement systems are now available. Permeable geotextile reinforcements may be especially useful because their drainage capabilities help to increase the structure stability by dissipating excess pore water pressures. Although reported results have led to some contradictory conclusions on the use of impermeable reinforcements, there is already strong experimental evidence that permeable inclusions can effectively reinforce poorly draining backfills (Zornberg and Mitchell, 1994; Mitchell and Zornberg, 1995).

Polymeric grid reinforcements provide adequate tensile strength required for the design of permanent reinforced soil structures. However, since they offer only limited in-plane drainage capacity, a low moisture content in the fill should be guaranteed by appropriate drainage systems throughout the design life of the structure. Geotextile materials with high in-plane hydraulic conductivity are reinforcements that offer the desired drainage capacity and are specially suitable for poorly draining fills. Particularly, composite geotextiles, which combine the hydraulic properties of nonwovens with the mechanical characteristics of geogrids or wovens, are probably the most appropriate reinforcement for marginal soils.

5 Backfill Requirements for Design of Reinforced Embankments

The purpose of these design recommendations is not to provide a new design procedure for geosynthetically reinforced slopes, but to evaluate current design practice for cases in which a decomposed granite backfill is used. The recommended guidelines in this report are based on current Federal Highway Administration guidelines (Christopher et al., 1989; Berg, 1992) and incorporate the findings of the centrifuge study performed as part of an extensive research program leading to this document (Zornberg et al., 1995). Where appropriate, recommendations specific to the use of decomposed granite as backfill material are indicated.

Determination of the foundation material is of particular importance in the case of decomposed granite. Soil profiles should be established below and behind the slope to a sufficient depth to evaluate potential for a deep seated failure. Foundation soil strength parameters, unit weight, and consolidation parameters should be established.

Properties of the available fill should then be obtained. Recommended backfill requirements for reinforced engineered slopes are (Christopher et al., 1989):

Sieve size	Percentage passing
4 in	100 - 75
No. 4	100 - 20
No. 40	0 - 60
No. 200	0 - 50

Plasticity Index (PI) is recommended to be less than 20, and magnesium sulfate soundness loss is recommended to be less than 30% after four cycles.

The maximum aggregate size should be limited to 3/4 inch (19 mm) for extensible reinforcements unless field tests have been or will be performed to evaluate potential strength reductions due to damage during construction.

The decomposed granite sampled for this study fits within the recommended backfill requirements indicated above. However, unique problems are associated with measuring the particle size distribution, specific gravity, and Atterberg Limits in decomposed granite.

The easily breakable nature of decomposed granite makes it difficult to establish a unique gradation curve. During the sieving process, aggregated particles may separate depending on the sieving load, the sieving time and the shaking level. Moreover, in decomposed granite, Atterberg Limits tests results may not be repeatable, because the finer portion of the soil sample may contain a high proportion of mica.

Figure 2 shows the grain size distribution for decomposed granite obtained from the Shasta County (Yapa et al., 1993). The apparent specific gravity for this material, Ga, is 2.577. The Atterberg Limits were determined on the portion passing the #40 sieve which represented less than 25 percent in weight of material. Both Liquid Limit and Plastic Limit were not determinable, and hence the material is considered non-plastic. Based on the classification system proposed by Lee and de Freitas (1989), this decomposed granite can be classified as Grade VI (residual soil). It can be further classified, based on Unified Soil Classification as SW-SM.

The backfill material should be compacted to at least 90% of optimum dry density according to AASHTO T-180 (modified Proctor) and the moisture content should be above w_{opt}-1%. The control of the moisture content during placement is of particular importance for the case of decomposed granite, since it is subject to hydrocompression if the soil is compacted dry of optimum (Yapa et al., 1993). Overall, the recommendation that cohesive soils be compacted in 6 to 8-inch compacted lifts and granular soils in 9 to 12-inch compacted lifts seems applicable in this case.

Peak shear strength parameters should be used in the analysis (Christopher et al., 1989). Parameters should be determined using direct shear or consolidated-drained (CD) triaxial tests. The peak angle of shear resistance for decomposed granite from Shasta County is indicated in Section 3 of this report. As can be seen from the data in Figures 5 and 7, the reduction in the peak friction angle ϕ_f with confining pressure is significant and should be considered. For other sites, site specific testing on local materials will be necessary to establish these parameters.

6 Performance Requirements

The factor of safety for slope stability should be adequate to address all uncertainties in the assumptions and design. Recommended minimum stability factors of safety, unless local codes require higher values, are (Christopher et al., 1989):

Sliding of the reinforced mass along its base, FS = 1.5

Deep seated failure, or external failure (overall stability), FS = 1.3

Compound failure (through reinforced zone), FS = 1.3

Dynamic loading, FS = 1.1

Against internal failure, FS=1.3

7 Allowable tensile strength

Limit equilibrium analysis assumes that the reinforcement and soil reach their design strengths at the same instant, regardless of deformation characteristics. A factor of safety is used to account for uncertainties in the strength of the geosynthetic reinforcements. Typical values for this factor range from $F_{s\text{-}u}$=1.3 to 1.5. The strength of the factored reinforcement should be available throughout the design life of the structure. To achieve this, partial safety factors for installation damage ($F_{s\text{-}id}$), creep ($F_{s\text{-}cr}$), and biological ($F_{s\text{-}bd}$) and chemical ($F_{s\text{-}cd}$) degradation should all multiply the already factored strength, so that geosynthetics possessing adequate strength, T_{ult}, could be selected. That is, the specified geosynthetic should have the following ultimate strength:

$$T_a = T_{ult} / (F_{s\text{-}u} \times F_{s\text{-}id} \times F_{s\text{-}cr} \times F_{s\text{-}bd} \times F_{s\text{-}cd})$$

Preliminary values for the partial safety factors in slope reinforcement are given by Koerner (1994):

	Geogrids	Geotextiles
$F_{s\text{-}id}$	1.1 to 1.4	1.1 to 1.5
$F_{s\text{-}cr}$	2.0 to 3.0	2.0 to 3.0
$F_{s\text{-}bd}$	1.0 to 1.3	1.0 to 1.3
$F_{s\text{-}cd}$	1.0 to 1.4	1.0 to 1.5

where:

T_s = Sum of required tensile force per unit with of reinforcement in all reinforcement layers intersecting the failure surface

M_D = driving moment about the center of the failure circle

D = Moment arm of T_s about the center of the failure circle (assume the radius of the circle for simplicity)

FS_R= Target minimum safety factor

FS_u= unreinforced safety factor

Use of design charts

The required total tensile force T_s can also be estimated using design charts. The different proposed design charts for reinforced soil slopes have similar characteristics: the desired overall soil factor of safety is accounted for by using a factored friction angle which, together with the angle of the slope, gives the required summation of reinforcement forces.

Leshchinsky and Boedeker (1989) use a logarithmic spiral failure mechanism to obtain the minimum factor of safety for reinforced slopes, while satisfying all three global limiting equilibrium equations. They assume that on the verge of failure the distribution of mobilized tensile resistance is linear with depth, proportional to the overburden pressure. Figure 8 shows the design chart for the required tensile force in the reinforcements. The procedure to obtain the total reinforcement requirements T_s is as follows:

1. Determine force coefficient Tm from the figure where $\phi_m = \tan^{-1} (\tan \phi / FS_R)$
2. Determine $T_s = 0.5\ Tm\ \gamma\ H^2$

where H is the total height of the slope.

Jewell (1991) presents an approach for the design of geosynthetically reinforced slopes using a two-part wedge analysis. The design chart is presented in Figure 9. The coefficient K_{req} in the design chart is equivalent to the normalized RTS value K, the parameter ϕ_d is the design friction angle of the backfill soil, and β is the angle of the reinforced slope. The procedure to obtain the reinforcement tension summation T_s as follows:

1. Determine force coefficient K_{req} from the figure where $\phi_d = \tan^{-1} (\tan \phi / FS_R)$
2. Determine $T_s = 0.5\ K_{req}\ \gamma\ H^2$

The use of design charts is based on the determination of a normalized Reinforcement Tension Summation (T_m coefficient in Leshchinsky's chart or K_{req} coefficient in Jewell's). This normalized value can be equally interpreted as an earth pressure coefficient that only depends on the soil strength and on the slope inclination. Experimental data obtained from the centrifuge testing performed as part of this study (Zornberg et al., 1995) are in agreement with this approach, since they consistently showed the validity of considering normalized coefficients: i.e. all reinforced embankment models built with the same slope and with the same backfill soil consistently yielded a unique value of the normalized Reinforcement Tension Summation.

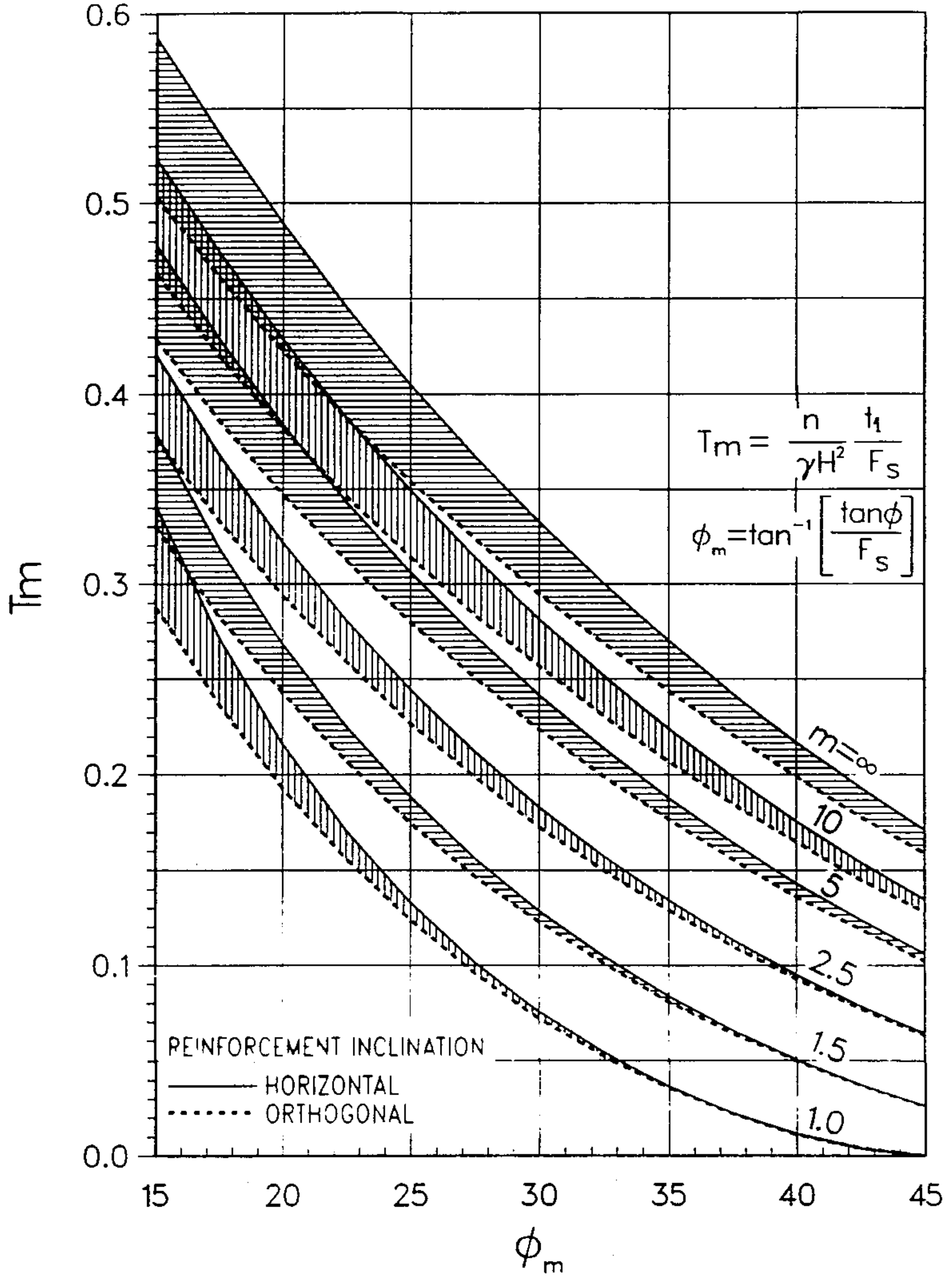


Figure 8 - Design chart for reinforced soil slopes (after Leshchinsky and Boedeker, 1989)

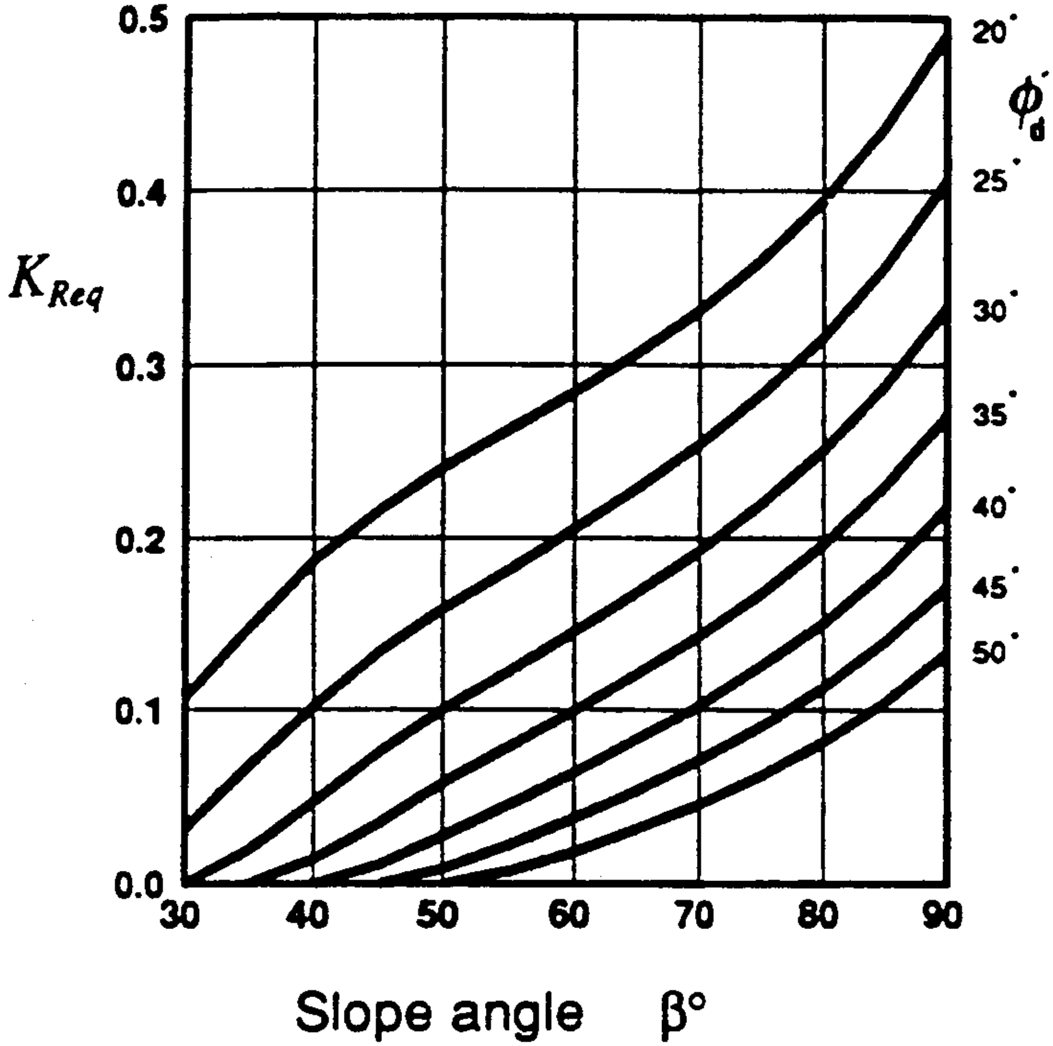


Figure 9 - Design chart for reinforced soil slopes (after Jewell, 1991)

8.2 Reinforcement Spacing and Length Requirements

A uniform vertical spacing is recommended. Although previous designs have considered a triangular reinforcement force distribution with depth, centrifuge tests (Zornberg et al., 1995) indicate that this distribution is not appropriate for reinforced slopes, and a uniform spacing is considered more appropriate for design purposes. This consideration is made for the case of slopes 0.5H:1V or flatter. The design tension T_d is then calculated by adopting a reinforcement vertical spacing S_v or, if the reinforcement strength is known, by calculating the minimum vertical spacing. The reinforcement spacing S_v and the design tension T_d are related by:

$$T_d = T_s \; S_v / H$$

where H is the structure total height.

The embedment length L_e of each reinforcement layer beyond the most critical sliding surface must be sufficient to provide adequate pullout resistance. The embedment length can be estimated by:

$$L_e = T_d \, FS / (2 \; F^* \, \sigma'_v)$$

where F* is the interaction factor (Christopher et al., 1989), and σ'_v is the vertical effective stress.

At the face, short (4- to 6- ft) lengths of intermediate reinforcement layers should be used to maintain a maximum vertical spacing of 2 ft for face stability and compaction quality.

8.3 Final detailed equilibrium analysis

Once a preliminary estimate of the total reinforcement requirements has been made, and a preliminary reinforcement layout (reinforcement spacing and reinforcement length) has been established, a rigorous limit equilibrium analysis should be performed. The preliminary evaluation discussed in the previous section is no substitute for this more rigorous analysis. This is because the preliminary evaluation has been done considering a simplified limit equilibrium method that only considers momentum equilibrium. Moreover, it is only with the limit equilibrium analysis of the final reinforcement layout that potential failure surfaces going beyond the reinforced zone can be evaluated.

Different analysis techniques are available to assess the potential for failure of a geotechnical structure. While the more rigorous plasticity solutions (limit analysis) or analyses that account for stress-strain behavior of soil and reinforcements (e.g., finite element analysis) have received increased attention, the limit equilibrium method (Terzaghi, 1956) still remains the most widely used approach to obtain approximate solutions for complex stability problems. This method assumes a failure surface and the stress distribution along that surface such that an overall equation of equilibrium in terms of stress resultants can be formulated (Figure 10).

Limit equilibrium analysis of unreinforced structures includes assumptions, such as the shape of the failure surface, that have to be made also in the analysis of reinforced soil slopes. Centrifuge test on reinforced slope modes have shown that the circular shape is appropriate for the analysis (Figure 11). Moreover, additional assumptions to those already introduced in the analysis of unreinforced structures are needed for the analysis of reinforced slopes. These include the inclination (e.g., horizontal, tangential) and distribution (e.g., linear, constant with depth) of the reinforcement tensile forces along the selected failure surface.

The limit equilibrium failure surfaces most widely used for the analysis of reinforced soil slopes include the planar wedge (Schlosser and Vidal, 1969; Lee et al., 1973; Segrestin, 1979), the bilinear wedge surface (Romstad et al., 1978; Stocker, 1979; Schneider and Holtz, 1986; Bonaparte and Schmertmann, 1987; Jewell, 1991), the logarithmic spiral (Juran and Schlosser, 1978; Leshchinsky and Reinschmidt, 1985; Leshchinsky and Boedeker, 1989), and the circular surface (Phan et al., 1979; Ingold, 1982; Bangratz and Gigan, 1984; Christopher and Holtz, 1985; Wright and Duncan, 1991). Several of these analysis methods have been used to develop design charts to determine the reinforcement requirements for simple slopes.

Although several different definitions for the factor of safety are currently being used, the definition considered in this study is given with respect to the shear strength of the soil:

$$FS = \frac{\text{Available soil shear strength}}{\text{Soil shear stress required for equilibrium}} \tag{1}$$

This definition is consistent with conventional limit equilibrium analysis, for which extensive experience has evolved for the analysis of unreinforced slopes.

A rigorous internal stability method should be selected for the analysis. Current design practices for design of reinforced soil slopes also often consider less rigorous analyses that decouple the soil-reinforcement interaction. Although they may be appropriate for preliminary analyses, they neglect the influence of reinforcement forces on the soil stresses along the potential failure surface, which may result in significantly different calculated factors of safety than those obtained using more rigorous approaches. Different rigorous methods of analysis have been developed for the analysis of geosynthetically reinforced slopes (e.g., Leshchinsky and Boedeker, 1989; Jewell, 1991; Wright and Duncan, 1991).

Computer programs have been developed for the limit equilibrium analysis of reinforced soil structures. These programs are based on conventional methods of slope stability analysis, adapted to consider forces provided by the reinforcements. Utexas 3.0 (Wright, 1990), which considers the Spencer's method for circular surfaces, is a flexible analysis tool oriented towards analysis rather than specifically for design. ReSlope (Leshchinsky, 1994) is an interactive program that considers log spirals for the potential failure surfaces, and allows the user to optimize the design. RSS (Christopher, 1994) is a limit equilibrium program being developed by Federal Highway Administration oriented towards an interactive design according to FHWA standards.

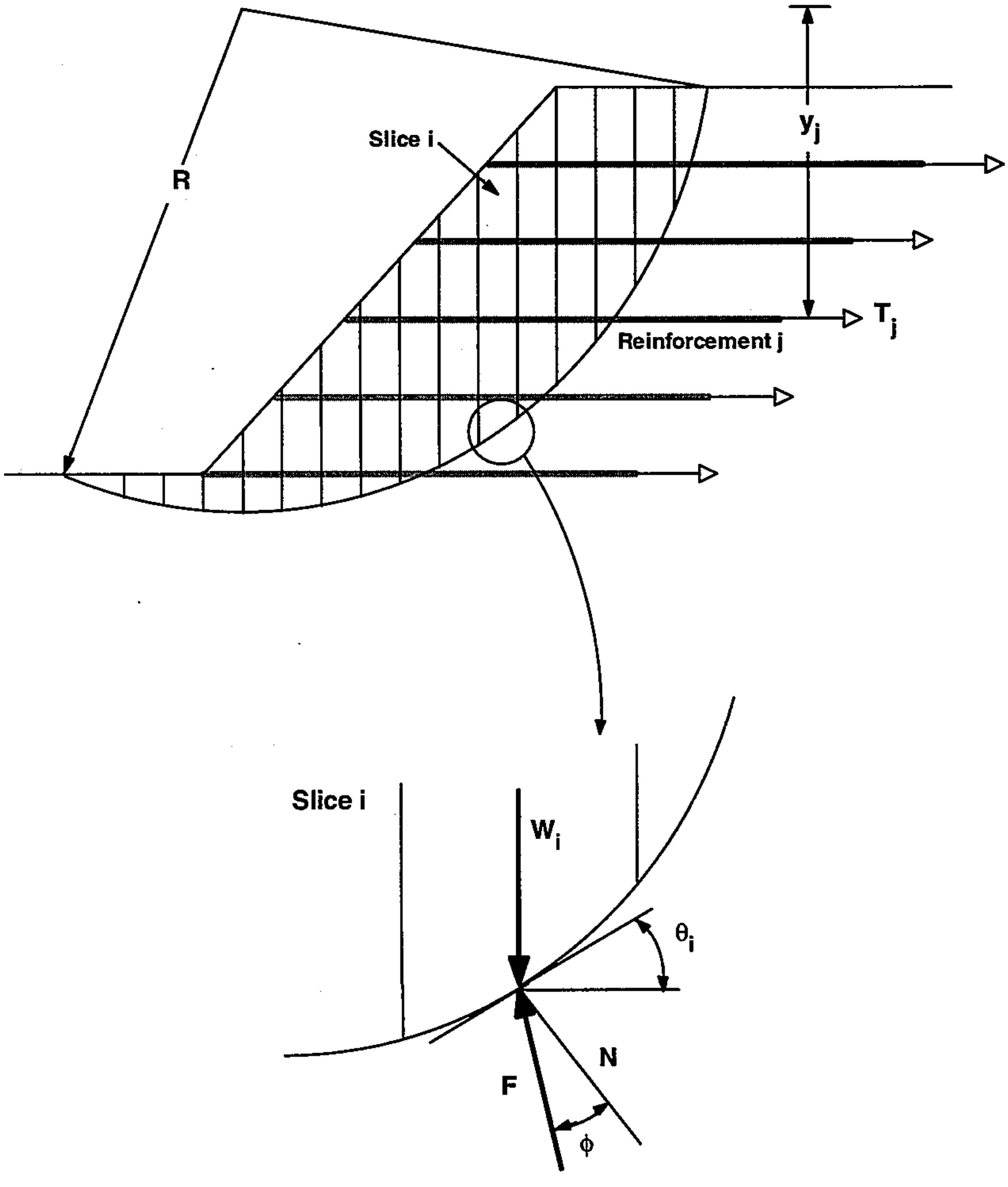


Figure 10 - Limit equilibrium of a reinforced soil slope using a circular failure surface

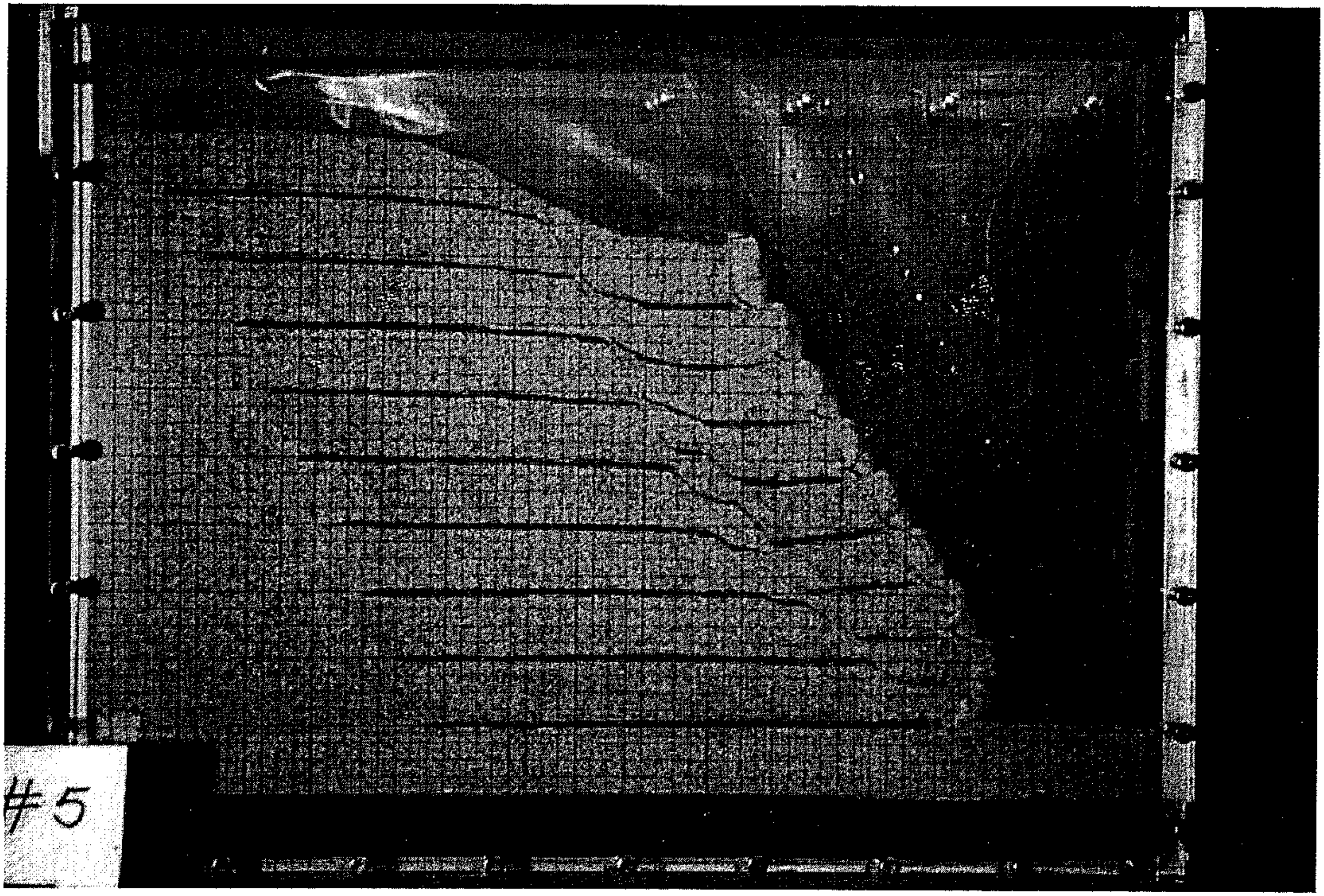


Figure 11 - Failure of a geotextile-reinforced slope model in a geotechnical centrifuge (after Zornberg et al., 1995)

9 Cost evaluation

Geosynthetically reinforced soil structures are nearly always more economical than conventional retaining wall alternatives. Savings are realized both from material and construction time reductions, and from reduced right-of-way requirements. The cost savings of a geosynthetically-reinforced slope over other grade separation options is illustrated in Figure 12 (Berg et al., 1990). The parameters used to develop this graph are as follows:

Reinforced Slope:

- Erosion protection: \$1.25 to \$4.00/yd^2
- On-site fill: \$2.00 to \$3.50/yd^3
- Reinforcement: : \$0.25 to \$9.15/vertical ft^2 of face

MSE Wall

- Select fill: \$5.00 to \$10.00/yd^3
- Facing panels, reinforcement, and design: \$17 to \$26/ft^2 of face

Cast-in-Place Concrete Cantilever Wall:

- On-site fill: \$2.00 to \$3.50/yd^3
- Reinforced concrete: \$30 to 50+/vertical ft^2 of face

In many highway applications, and mainly in those involving widening of existing ways, the alternative designs under consideration are geosynthetically reinforced slopes and flat nonreinforced slopes. While the reinforced soil alternative should account for the cost of the geosynthetic inclusions, the following economic benefits should be considered in comparison to a flatter slope:

- Savings in backfill material
- Savings in excavation, resulting from selection of an alignment that accounts for the lower volume of required backfill material
- Savings in erosion control matting per vertical ft^2 of face, given the lower total face area. If wrap around geotextiles are used in the facing of the structure, additional cost savings can be achieved in this item, and are expected to perform better than matting for erosion control.
- Savings originated from the lower length of the culverts
- Savings from the reduced required rights-of-way
- Lower construction time
- Lesser disturbed area, possibly reducing environmental impact and mitigation

As an example, a cost evaluation is presented in the next section for a case history of a geotextile reinforced embankment that used decomposed granite as backfill material.

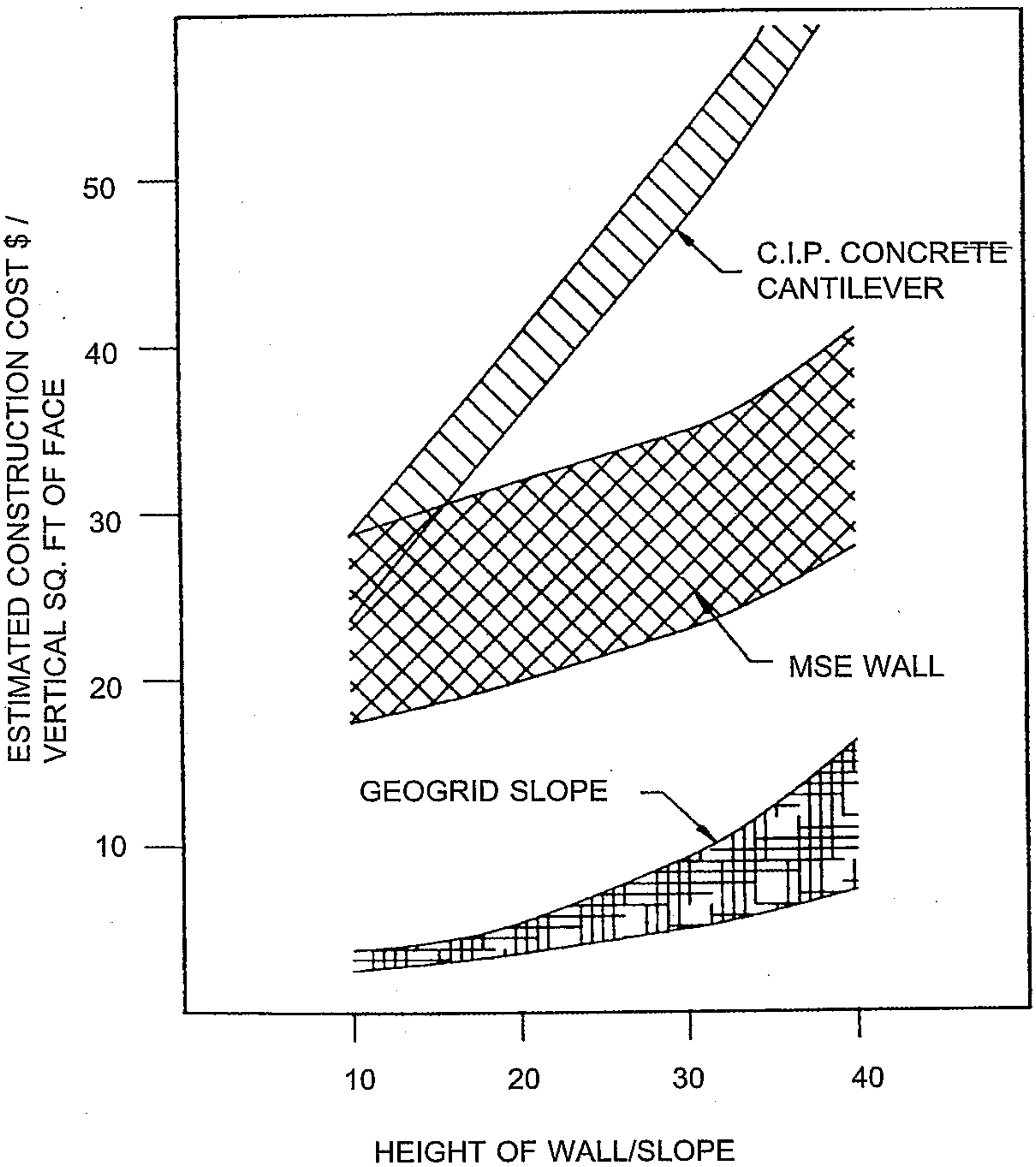


Figure 12 - Estimated construction costs for retaining structures (after Berg et al., 1990)

10 Case History

The project consists of a geotextile-reinforced slope designed as part of the widening of U. S. Highway 93 between Salmon, Idaho, and the Montana state line (Zornberg, 1994). The reinforced structure is a 1H:1V (45°) slope located in Idaho's Salmon National Forest along Highway 93. Esthetics was an important consideration in the selection of the retaining structures along scenic Highway 93, which has been recognized by a recent article in National Geographic (Parfit, 1992). The 172 m long and up to 15.3 m high geotextile-reinforced slope is vegetated, causing a minimum environmental impact to the Salmon National Forest. This structure was designed by the Western Federal Lands Highway Division (Barrows and Lofgren, 1993), and represents one of the highest geotextile-reinforced slopes in the U.S.

The slope was designed using geotextile reinforcements that not only were required to have adequate tensile strength, but were also expected to provide appropriate in-plane drainage capacity to allow dissipation of pore water pressures that could be generated in the fill. In this way, an additional drainage system was not necessary even though indigenous soils were used as backfill and groundwater seeping was expected from the excavation behind the fill. Due to the unique characteristics of this structure, the reinforced slope was considered experimental, and an extensive program of instrumentation and construction monitoring was implemented to evaluate its performance.

10.1 Design Considerations

Use of decomposed granite as backfill material. On-site soil coming from excavation of the road alignment was to be used as backfill material. Subsurface drilling revealed that the majority of subsurface material on this project is granitic in origin, ranging from hard, intact rock to highly decomposed, soil-like material. Preconstruction evaluation of the soil in the proposed cut and borrow area indicated a maximum density of 18 to 21 kN/m^3 and an optimum moisture content of 9.5 to 13.5%, as determined by Standard Proctor tests. Although the project specifications required the use of material with no more than 15% passing U.S. No. 200 sieve, internal drainage was a design concern. This was because of the potential seepage from the fractured rock mass into the reinforced fill, especially during spring thaw, coupled with the potential crushing of decomposed granite particles that could reduce the hydraulic conductivity of the fill.

Design methodology. Design of the geotextile-reinforced slope, done according to FHWA guidelines, included analysis of the external and internal stability (Christopher et al., 1989). The external stability was evaluated by analyzing the potential for sliding and for overall deep-seated slope failure. Since a detailed subsurface investigation revealed low-strength decomposed granite zones, a reinforced rock shear key was built at the base of the reinforced slope in order to increase the factor of safety against a failure outside the reinforced zone or a failure partially through the reinforced and unreinforced zones, i.e. to increase both external and compound stability. Methods of slope stability analysis, adapted to consider forces provided by the reinforcements, were used to determine the required geotextile layer spacing and reinforcement tensile strength. The total reinforcement length that provides adequate pullout resistance was finally calculated. The selected geotextiles were evaluated by performing product specific creep tests and a construction damage assessment (Wayne and Barrows, 1994). The results were used to develop the partial factors of safety that estimate the geotextile allowable tensile strength.

Reinforcement layout. Widening of the original road was achieved by turning the existing 2H:1V nonreinforced slope into a 1H:1V reinforced slope. The specified geotextile strength was varied with the height of the slope to more closely match theoretical design strength requirements. As shown in Figure 13, the final design adopted two geosynthetically reinforced zones with a constant reinforcement spacing of 0.3 m (1 ft). At the highest cross-section of the structure, the reinforced slope has a total of 50 geotextile layers. A nonwoven geotextile (PP-20) was selected in the upper half of the slope, while a high strength composite geotextile (PPC-100) was used in the lower half. Both selected geosynthetic reinforcements were manufactured by Polyfelt. The PP-20 material, with an ultimate tensile strength of over 20 kN/m, is a polypropylene continuous filament needle punched nonwoven. The PPC-100, with an ultimate tensile strength over 100 kN/m, is a polypropylene continuous filament nonwoven geotextile reinforced by a biaxial network of high-modulus yarns. Both materials exhibit a typical in-plane hydraulic transmissivity of 0.006 l/s/m under 200 kPa of normal stress. The composite geotextile was chosen for the lower half of the slope given the design need of combining the reinforcing benefits of high-modulus geosynthetics and the hydraulic advantages of nonwovens.

Basis for geosynthetic selection. The decision to use a reinforced soil slope was based on the ease of construction, the anticipated lower cost as compared to more conventional structures, and the reduced environmental impact of this solution. On the other hand, the use of reinforcements with appropriate in-plane transmissivity was specified in order to deal with potential seepage from the fractured rock mass. The lateral drainage provided by the reinforcements would avoid the need of a separate drainage system. There are no general design guidelines for reinforced soil structures built with poorly draining soils. Nevertheless, since several reinforced structures of this type have already been constructed, many lessons can be learned from past experience. Permeable geosynthetics were specified for this FHWA project based on the experimental evidence that these reinforcements can more effectively reinforce poorly draining soils.

10.2 Construction

Slope construction, performed using conventional construction equipment, took place during the summer of 1993. The original slope was excavated back to a 1H:1V side slope, and the base for the embankment was graded to a smooth condition. The rock shear key was constructed by depositing, spreading, and then compacting the rock material with a vibratory roller (Figure 14). The rock shear key was reinforced with welded wire mesh having a vertical spacing of 0.45 m. The selection of a welded wire mesh reinforcement was based on the large openings required to accommodate the size of the rock material in the shear key (up to 380 mm). Although construction took place during the dry summer season, seepage appearing as weeps at the base of the cut slope emerged from the fractured rock mass.

No special expertise was required for slope construction, and a crew of five members without previous experience in reinforced soil construction placed an average of three layers per day along the instrumented, 172 m long slope. In each lift, backfill material was spread with a medium sized bulldozer and oversized rocks (greater than 100 mm) were then removed. Each layer had to be compacted to 95% of maximum density, as determined by Standard Proctor tests, and the water content of the backfill was specified to be within 3% of the optimum. These compaction requirements were easily achieved by the contractor using static compaction methods: a grid roller was used for compaction of most of the fill, and a small walk-behind compactor was used close to the facing. Figure 15 shows the compaction equipment typically used throughout construction of the geotextile-reinforced slope. Special care was required when working around the inclinometer tubes during slope construction. The geotextile at each lift level was placed with the machine direction perpendicular to the slope, overlapping adjacent rolls a minimum of 0.60 m. Although initial design did not consider wrapping of the geotextiles at the slope face, the geotextiles were eventually wrapped in order to satisfy National Forest Service requirements. A single layer forming system inclined 45° was used, and holes were made through the geotextile on the face to permit vegetation.

Placement of the top layer (layer 50), was finished approximately one month after placement of the initial layer. An erosion control matting was subsequently placed on the slope and anchored to protect the face until vegetation is well established. Figure 16 is a view of the completed geotextile-reinforced slope after the erosion control matting has been placed. The subgrade was completed in the 1993 summer season and the reinforced slope has performed as intended since then. A considerable amount of instrumentation data has been accumulated during the construction period, and post-construction performance is still being monitored at this writing. The results from the instrumentation program reveal an excellent performance of the slope to date: small deflections and strains in the reinforcements, as well as negligible overall post-construction movements.

10.3 Costs

The bids for the geotextile reinforced embankment considered measurements by the square foot of embankment face constructed. Measurement was made on the plane of the front face, including the rock shear key face. Since the reinforced embankment was heavily instrumented, special care was exercised in the selection of the contractor for the construction of the geotextile-reinforced embankment. The selected bid resulted in a cost of $14.00 per square foot of face, and was chosen from a pool of eleven bids with an average unit price bid of $11.5 per square foot of face. This cost includes the backfill material (approximately $\$4.00/yd^3$ of decomposed granite), the excavation (approximately $\$3.50/yd^3$ of excavation), and the geotextile reinforcements (between $1.50 and $\$3.50/yd^2$). The erosion control matting was bid separately (with an average bid of $\$3.00/yd^2$ on the ground surface).

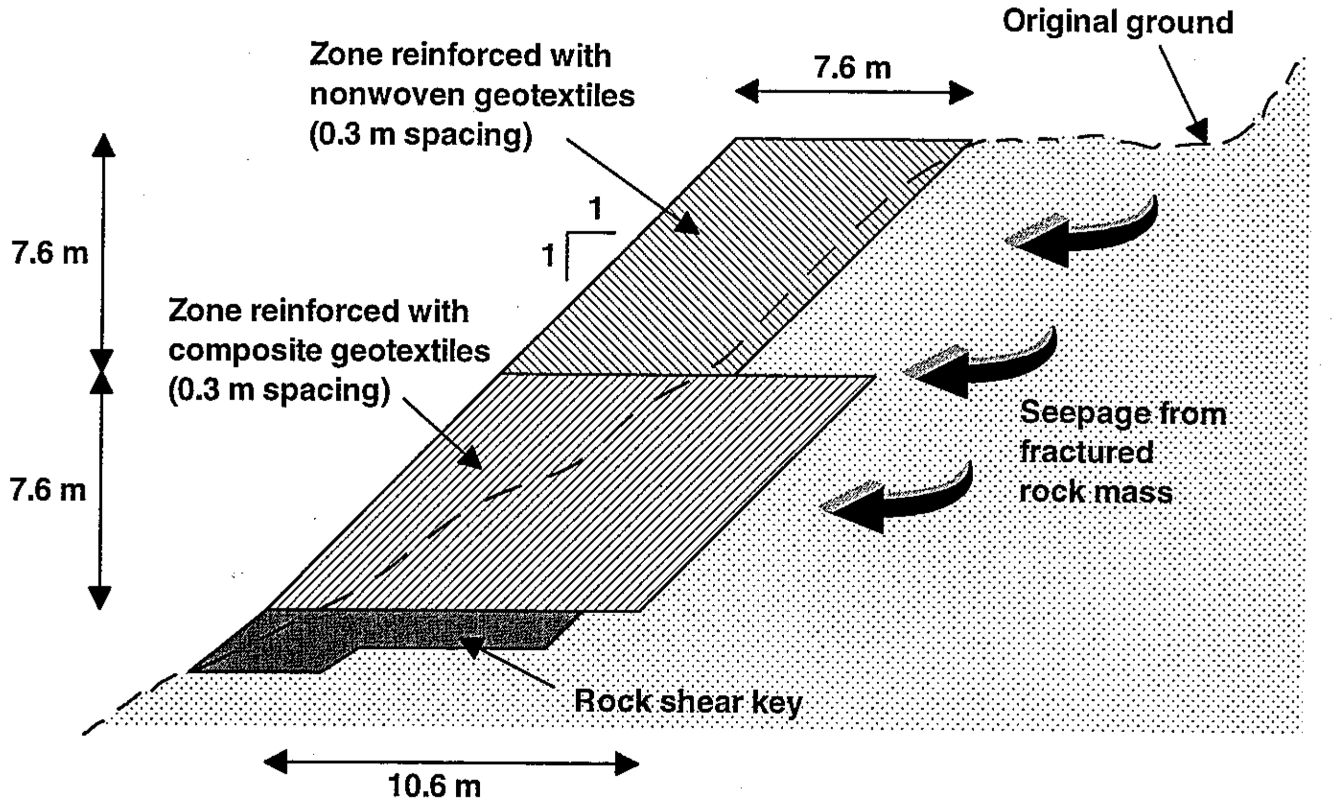


Figure 13 - Cross section of the geotextile-reinforced slope (after Zornberg, 1994)

Figure 14 - Rock shear key under construction

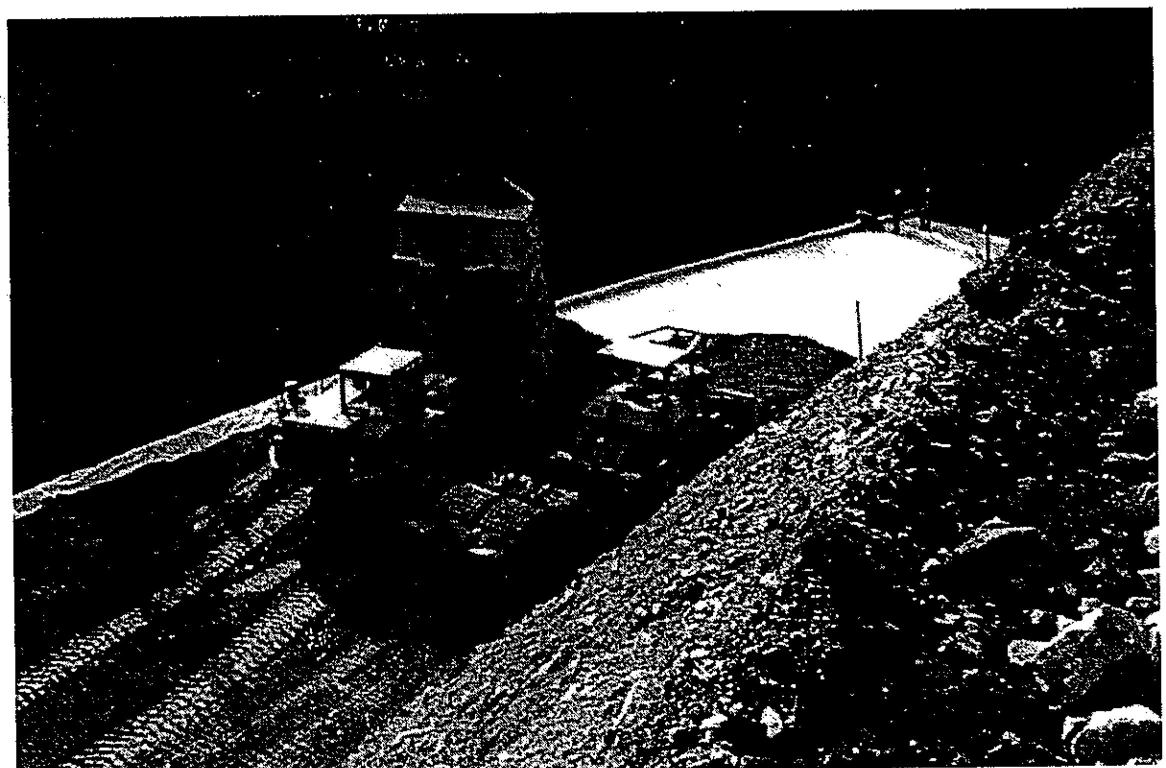


Figure 15 - Top view of the reinforced slope during compaction operations

Figure 16 - View of a 15.3 m high geotextile-reinforced slope built using decomposed granite as backfill material

11 Conclusions

This report provides a summary overview of the results of a study of the feasibility of using geosynthetic reinforcements in construction of embankments with decomposed granite backfill. Although based on its grain size distribution decomposed granite may satisfy current backfill standards for geosynthetically reinforced embankments, some considerations should be made to accommodate the special character of this material (Yapa et al., 1993). The friction angle of compacted decomposed granite decreases significantly with increasing stress level.

Centrifuge testing has provided much needed evidence that limit equilibrium methodologies adequately predict the performance of geosynthetically reinforced soil structures at failure (Zornberg et al., 1995). Although these results validate current design practices in the United States (e.g., Christopher et al., 1989), refinements to current design procedures are suggested when appropriate based on the findings of the centrifuge study. The centrifuge study showed that all reinforced embankment models built with the same slope and with the same backfill soil consistently yielded a unique normalized Reinforcement Tension Summation. This is in agreement with current chart design procedures that are based on the use of normalized coefficients to determine reinforcement requirements.

Cost evaluation for the construction of geosynthetically reinforced slopes shows that they compare favorably with other types of slope construction. A case history of a permanent reinforced embankment built using decomposed granite as backfill material shows that the construction can be rapid and can be readily achieved with regular field equipment. Moreover, the resulting structure meets both aesthetic and structural integrity criteria. Thus, overall, there seem to be no technical impediments to the use of decomposed granite materials in the construction of geosynthetically reinforced embankments.

12 Implementation

The methodologies and guidelines for the design of geosynthetically reinforced slopes using decomposed granite backfill presented herein and in preceding reports, Yapa et al. (1993) and Zornberg et al. (1995), are based on an evaluation of materials from the Shasta Bally batholith and on an analysis and review of current practice. As such these reports are intended to provide general guidance in terms of design considerations and expected performance. They also provide a baseline reference for expected behavior of decomposed granite soils. Thus, the results and recommendations should be applicable to other sites underlain by decomposed granite for the purposes of preliminary feasibility assessment.

The actual reinforced embankment designs in the region underlain by the Shasta Bally batholith will have to be developed by the appropriate geotechnical function as District 2 of the California Department of Transportation identifies specific sites and projects. Similarly, site specific designs based on the actual material properties at the site will have to be developed for application in other regions.

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